NMG
Geotechnical, Inc.

August 15, 2011

Project No. 10028-04

To: Shea Homes
1250 Corona Pointe Court, Suite 600
Corona, California 92879

Attention: Mr. John Danvers

Subject: Geotechnical Review of the Western Edge of the Proposed Baker Ranch Development, Including Borrego Wash Improvements and the Bypass Channel, Tentative Tract 16466, Baker Ranch, Lake Forest, Orange County, California

In accordance with your authorization, NMG Geotechnical, Inc. (NMG) has reviewed the improvement plans for the Borrego Wash Canyon Bypass Channel, prepared by Hunsaker and Associates (Hunsaker) and conducted subsurface exploration in conjunction with our geotechnical review of the western edge of the planned Baker Ranch Development. The site is located in northwestern Lake Forest, California (Figure 1). This portion of the site is approximately 50 acres in size and is located adjacent to Borrego Wash, west of Alton Parkway and south of SR 241.

The primary purpose of NMG's work was to evaluate the proposed improvements along the western perimeter of the property in light of the existing geotechnical conditions. This study included review of prior geotechnical borings, cone penetrometer test (CPT) soundings, and geologic mapping by Pacific Soils, Inc., as well as supplemental subsurface exploration conducted by NMG. Our field work consisted of site reconnaissance, geologic mapping, advancement of seven CPT soundings and excavation of eight hollow-stem-auger borings. Laboratory testing and geotechnical analysis was conducted specifically to evaluate slope stability, settlement, hydro-consolidation, and seismic hazards including liquefaction, lateral spread and seismic settlement. We have also evaluated the potential erosional impacts of Borrego Wash to the proposed development.

This report also addresses the geotechnical engineering review letter from the City of Lake Forest dated May 23, 2011 prepared by Wildan Geotechnical (a copy of which is provided at the rear of text). The focus of their review comments are related to potential liquefaction, lateral movements and settlement of the proposed slopes and improvements along Borrego Wash.

Primary geotechnical constraints we have identified and analyzed along with our recommended remedial measures (in parentheses) include:

1. Settlement of fill placed over alluvium to be left in place (20- to 40-foot removals down to saturated alluvium, enhanced fill compaction, and settlement monitoring),

2. Settlement of the box culvert under the influence of proposed fill above and adjacent the culvert (placement of fill to design grades in a 500-foot section of box prior to box construction); and
3. Potential for post-liquefaction lateral flow failure of slopes immediately adjacent to the wash (excavation of a 40-foot-wide, 12- to 15-foot-deep shear key below the remedial removal bottom).

Based on our study, the proposed grading is considered geotechnically feasibly, provided the recommendations of this report are implemented during design, grading, and construction. It is our opinion that the bypass channel and planned grading provide substantial improvement to the active Borrego Wash.

This report presents our updated findings, conclusions, and recommendations for the proposed grading and construction.

If you have any questions regarding this report, please contact our office. We appreciate the opportunity to provide our services.

Respectfully submitted,

NMG GEOTECHNICAL, INC.

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1.0 INTRODUCTION

1.1 Introduction and Purpose

NMG Geotechnical, Inc. (NMG) has reviewed the improvement plans for the Borrego Wash Canyon Bypass Channel, prepared by Hunsaker and Associates (Hunsaker) for the proposed western perimeter of the Baker Ranch Property, Tentative Tract Map 16466, in the City of Lake Forest, California. A tentative tract plan was previously reviewed by NMG which evaluated the overall property and provided conclusions and recommendations for the proposed development (NMG, 2011). The City of Lake Forest's geotechnical consultant, Wildan Geotechnical (Wildan), reviewed the prior report and issued a geotechnical engineering review letter dated May 23, 2011 (included at the rear of text). The purpose of this study was to evaluate the proposed Borrego Wash improvements along the western edge of the development in light of the existing geotechnical conditions and to address the review comments by Wildan. The improvement plans, prepared by Hunsaker, received by NMG on August 4, 2011 were reviewed for this study and were used as the base map for the 40-scale Geotechnical Map in this report (Plates 1 through 4).

1.2 Scope of Work

The scope of work for this study included the following tasks:

- **Background Research:** Review of available geotechnical reports, maps and stereoscopic aerial photographs dating back to the 1950s was performed. References and aerial photos reviewed are listed in Appendix A.
- **Compilation of Existing Data:** Data obtained from the prior investigations at and adjacent to the site were compiled and shown on the Geotechnical Maps. Pertinent boring logs and laboratory testing data are included in Appendices B and C.
- **Site Reconnaissance and Geologic Mapping:** Site reconnaissance was performed to review the existing geotechnical conditions and mark the proposed CPT and hollow-stem-auger boring locations. Underground Service Alert (USA) was notified and NMG coordinated with nursery personnel to clear the boring locations. Geologic mapping was performed in the northerly portion of Borrego Wash, where bedrock is exposed.
- **Subsurface Field Exploration:** NMG conducted a supplemental subsurface field exploration along the western perimeter of the site in July, 2011. The exploration consisted of advancement of seven additional Cone Penetrometer Tests (NCPT 1 through NCPT 7) with two seismic refraction studies and excavation, visual logging and soil sampling of eight additional hollow-stem-auger borings (H-1 through H-8).
- **Laboratory Testing:** Laboratory testing was performed on selected soil samples collected from the borings and trenches. Results of these tests are included in Appendix C. Pertinent laboratory test results from the prior geotechnical investigations by NMG and others were also reviewed and are included in Appendix C.
- **Plan Review and Geotechnical Analysis:** Geologic cross-sections were prepared based on the compiled data and the updated improvement profiles. This data is presented on the Geotechnical Map (Plates 1 through 4) and Cross-Sections A-A', through F-F' (Plates 5 and

6). The proposed improvements were reviewed in light of the collected data. Geotechnical analysis was performed to evaluate liquefaction, settlement, slope stability, earthwork and constructability and provide recommendations for grading and implementation of the proposed improvements.

- **Report Preparation:** Preparation of this geotechnical report with the accompanying illustrations and appendices. This report summarizes our updated findings, conclusions, and recommendations for the planned grading and provides preliminary design information for the future site development.

1.3 Site Location and Conditions

The overall development is approximately 387± acres in size and located in the city of Lake Forest, adjacent to the former El Toro Marine Base and southwest of the Foothill Transportation Corridor. The subject portion of the site is approximately 50 acres in size and is located adjacent to Borrego Wash along the westerly perimeter of the overall development. To the west of the site (adjacent to the wash), the site is bordered by the former El Toro Marine Corps Air Station.

The site is within the foothills of the Santa Ana Mountains. It consists of the gently sloping Borrego Wash and an elevated alluvial terrace plain adjacent to the channel. The channel walls between the wash and the terrace range from 15 to 35 feet high and have slope angels that from near vertical to approximately 1H:1V. The terrace area is currently being used as a nursery and contains a network of dirt roads, some overhead electric lines and numerous plant and tree containers, above ground irrigation lines and fences/gates.

1.4 Site Historical Conditions

Historical stereographic aerial photographs dating from 1952 through 2009 were reviewed to determine past uses and conditions at the site. The following are the major highlights of this aerial photographic review based upon the photographs referenced in Appendix A.

- Up to 1952, the site appears relatively untouched by human activities except for a few minor dirt roads that cross the site. The Borrego Wash's active channel consists of a braided stream that randomly meanders across the flood plain. The hills are covered with grasses, light brush and a few trees. The site remains in a similar condition throughout the 1950s.

- By 1965, the northwestern portion of the site was being prepared for a citrus orchard. These activities consisted of grading the active channel of Borrego Wash to "match" the rest of the flood plain. To do this, a narrow and straight channel was excavated along the northwestern boundary with the El Toro Marine base. A row of eucalyptus trees was also planted along this boundary on the northwestern side of the channel. In 1965 the citrus trees were newly planted. The disturbed area was limited primarily to the Borrego flood plain.

- Sometime between 1965 and 1967, the citrus orchards had expanded to fill in most of the canyon areas and the Borrego Plain. They appeared to be large enough to produce fruit.

- Throughout the 1970s, the citrus orchards appeared to be in full operation with little or no changes. The various windrows of eucalyptus trees had matured to form wind blocks.

- Up until 1988 the condition of the subject site remained relatively unchanged. The orchard appeared to be active and well maintained, and the active Borrego channel was still along the northwestern border.

- The 1990s brought significant changes to the entire Lake Forest and Foothill Ranch area to the north. By 1992, Bake Parkway and the area on the southeast border had been graded. Foothill Ranch to the north of the future toll road had been mostly graded and some homes were built and occupied. Most of Baker Ranch was occupied by the same orchard operation and the Borrego channel was still a narrow ditch along the northwestern boundary with a row of parallel eucalyptus trees. The toll road to the north started construction in (or prior to) 1992 and, by early 1993, was mostly graded, although not completed. Approximately half of the homes in Foothill Ranch had been constructed and the commercial and residential projects immediately north of the toll road were being graded. Grading of the toll road created a channel beneath the road that collected the upstream waters and funneled them to the northwestern corner of the property. A trapezoidal debris basin, which was present in early 1992 in this area, was gone in 1993. In early 1993 the active Borrego channel appears to have been widened by erosion, including the loss of many eucalyptus trees. A plume of new sand can be observed to have been recently deposited just beyond the property line in the southwest corner.

- Throughout the 1990s the development of Foothill Ranch properties to the north and Lake Forest properties to the east and southeast continued. The citrus orchard operation also continued during this time period until approximately 1997, when the trees were removed in the Borrego Plain and this area was converted to a nursery operation. The Borrego channel continued to widen throughout the 1990s. By 1999, most of the eucalyptus trees along the southwestern border were no longer in place due to bank erosion, and erosion of the channel wall increased towards the southeast.

- From 2000 to present, the nursery operation has continued, but the orchards have been removed. The active Borrego channel is now up to 30 feet in depth, below the plain, and has widened from less than 20 feet wide (1959 to 1992) to greater than 125 feet wide after the winter storms this year (2011).

1.5 Previous Geotechnical Investigations

The site was originally studied by the California Division of Mines and Geology (CDMG) in 1973 and later in more detail by the U.S. Geological Survey (USGS) and CDMG in 1981. The CDMG published their engineering geology report that includes this site, in 1984 (references). The project site has been the subject of prior geotechnical studies for the planned Baker Ranch development and the adjacent section of Alton Parkway that extends through Baker Ranch and to Irvine Blvd.

- Baker Ranch was the subject of a preliminary geotechnical investigation and tentative tract map review by Pacific Soils Engineering (PSE, 2002) addressing a prior development plan.

That study forms the primary database for this study. Site-specific boring and trench logs, and laboratory testing results conducted for that study are included in Appendices B and C of this report. The PSE (2002) study used information gleaned from many previous PSE reports. Boring and trench logs and laboratory test results from those studies are also included in the appendices. Boring and trench logs and laboratory tests results are separated in the appendices based upon the date that they were performed. Prior to PSE's 2002 study, several other geotechnical investigations and grading operations have been conducted onsite and on surrounding properties.

- Kleinfelder (2009) conducted a geotechnical review of the portion of Alton Parkway that extends from Commercentre Drive to the southwest of Baker Ranch. The grading of this portion of the roadway also commenced in late 2010 with Kleinfelder as the consultant of record. This grading and utility construction project is anticipated to be completed in early 2012. This project includes construction of the intersection of Commercentre and Alton Parkway which is within the Baker Ranch property boundary.

- In 2010, Hushmand and Associates, Inc. (HA) conducted a geotechnical investigation of the proposed alignment of Alton Parkway through the Baker Ranch property. In late 2010 grading began on this roadway and is currently ongoing. HA is the consultant of record during this grading for the City of Lake Forest and NMG is providing a second-party review of the grading operations. Grading aspects of the roadway are anticipated to be completed in 2011.

1.6 Proposed Improvements

The western edge of the planned Baker Ranch Development will consist of residential property at the top of a slope overlooking the remaining Borrego Wash area. The building pad elevations range from 620± feet at the south end (adjacent to the storm drain outlet structure) to 685± feet at the north end. The overall slope descending to Borrego Wash is approximately 50 to 55 feet high with a 35- to 40-foot-wide mid-slope bench for an access road/hiking trail over the box culvert. The upper slope ranges from 25 to 30 feet high along the entire length. The lower slope ranges from 20 to 25 feet high from Station 20+00 to 41+00. North of Station 41+00 the lower slope is variable, with over-steepened soil cement slopes noted on the plans.

The proposed storm drain improvements for control of the surface drainage for Borrego Wash consist of intercepting the majority of the water flow from the channel at the north end of the property and transmitting it into a 12- to 18-foot-wide by 10-foot-high reinforced concrete box (RCB) that extends along the western edge of the development. The water would be intercepted by an inlet structure adjacent to the existing outlet structure at the north end of the site and transmitting the main flow into the RCB. Low flows will be intercepted by a 60-inch reinforced concrete pipe (RCP) at the existing outlet structure north of the subject site and channeled to the existing open drainage course. The low flow water pathway would extend along the project boundary.

A 35-foot easement/trail is planned directly over the RCB, with manufactured slopes descending to the active Borrego Wash to the west and ascending to the future development to the east. Design fill slopes descending to Borrego Wash are up to 30 feet high and range from 0.3H:1V to

2H:1V, with the portions steeper than 2H:1V (approximate RCB Stations 41+50 to 52+00) to be constructed with soil cement. The design fill slopes ascending up to the future development are up to 30 feet high and designed at 2H:1V. The proposed preliminary grading will involve minor design cuts on the order of 10 feet deep and fills up to approximately 40 feet thick.

2.0 GEOTECHNICAL FINDINGS

2.1 Regional Geologic Setting

The project site is located within the Peninsular Range Province, a geomorphic province with a long, active geologic history, including shallow marine deposition followed by uplift and both fluvial and marine erosional processes. The site is within the southeastern margins of the late Miocene-age Los Angeles Basin and lies in the southwestern foothills of the Santa Ana Mountains. Sandstone of the Capistrano Formation underlies the site, which was formed during deposition of the near-shore portion of the Capistrano Embayment. Approximately four million years ago, the tectonic boundary shifted to the San Andreas Fault system, creating compression and uplift of the embayment. To the north, this region is bounded by the Whittier-Elsinore fault zone (located 19 kilometers north of the site), and to the south, by the Newport-Inglewood fault zone (located 21 kilometers south of the site).

2.2 Earth Units

The site is underlain by bedrock of the Tertiary Capistrano Formation (Oso Member). Quaternary terrace deposits cap the lower lying ridges and Quaternary alluvium and colluvium have in-filled the ancient channels. Minor undocumented fills associated with the nursery operations are locally present throughout the site. These earth units are depicted on the accompanying geotechnical maps (Plates 1 through 4).

Capistrano Formation, Oso Member (Map Symbol - Tco): This bedrock was deposited in a shallow marine environment during the Tertiary Period approximately 1.5 to 5 million years ago. It is exposed in portions of the existing Borrego Wash and underlies the entire site. All our hollow-stem-auger borings encountered this bedrock unit at the bottom of the excavations. The material consists of white, olive gray to dark grayish brown, and light to dark gray silty and clayey fine to coarse sandstone, which is moist to wet, very dense to hard, micaceous and friable. A few geotechnical borings by others encountered mica rich clay, or bentonitic clay beds. The material is generally massive to poorly-bedded. Bedding, highlighted by lenses with larger grain-size, is exposed in the existing wash near the northern portion of the site.

Quaternary Terrace Deposits (Map Symbol - Qt): Terrace deposits are present east of the subject area. These deposits represent the dissected remnants of the former flood plain/stream bed, produced during an earlier stage of erosion and deposition. The terrace deposits are typically tan/reddish brown, silty/clayey sands with occasional pebble and cobble lenses. The material ranges from loose near the surface to dense at depth and dry to moist. Much of the terrace deposits were derived from the bedrock units in the Santa Ana Mountains to the northeast.

Alluvium/Colluvium Undifferentiated (Map Symbol – Qac): Quaternary-age alluvial/ colluvial deposits are found beneath the majority of the subject site. Due to similar engineering characteristics and for ease of discussion, alluvium and colluvium were undifferentiated by PSE. These sediments originated from the surrounding bedrock and terrace deposits units and have been transported by water and/or gravity. Due to the consistent character of the bedrock unit, the alluvium/colluvium is fairly uniform and, based on our geotechnical borings, consists of light brown to yellowish brown poorly graded sand, silty sand, and locally clayey sand and sandy gravel. The deposits were found to be damp to wet, loose to medium dense and highly friable. Locally, at the contact with the underlying sandstone bedrock, a basal gravel layer was encountered. Alluvial deposits within the Borrego flood plain reach depths in excess of 70 feet as indicated in the boring logs (Appendix B).

Artificial Fill (Map symbol – Afu): Areas of undocumented artificial fill **(Afu)** occur locally across the site, generally associated with past agricultural and/or nursery activities, including deep plow zones, access roads, in-filled old drainage channels, and irrigation/water lines. During development of the citrus grove, the active channel of the Borrego Wash was filled-in and redirected to the northwestern boundary. The depth or nature of the fill in this prior channel is uncertain. However, in general, these and other fills are likely derived from onsite soils and bedrock materials and consist of loosely compacted silty to clayey sands, with varying amounts of debris. These fill material were not tested nor were unsuitable earth materials below these fills documented and are subject to removal.

2.3 Geologic Structure and Faulting

The general overall geologic structure within the site consists of a homoclinal sequence where bedding is generally dipping to the west and southwest. Local variations are apparent due to cross bedding and paleo-erosional surfaces. Morton and Miller (1976 and 1981) mapped the contact between the Monterey Formation and Capistrano Formation as a fault. Publications/reports covering this area indicated this fault is not active.

The alluvium and terrace deposits are generally flat lying, with a gentle dip toward the southwest (down-gradient).

2.4 Seismicity and Seismic Hazard Zones

Faulting: The site is not located within a fault-rupture hazard zone as defined by the Alquist-Priolo Special Studies Zones Act (CDMG, 1999). There are no known major or active faults mapped within the proposed development area, and no evidence of active faulting was observed during prior work at the site (Appendix A). Our recent site exploration, past investigations and geologic mapping during rough grading at the site and adjacent areas did not encounter geomorphic expressions or visible lineaments associated with active faulting at the site. Therefore, the potential for primary ground rupture at the site is considered slight to nil.

Using the USGS computer program (2002, updated 2008) and the site coordinates of 33.6743 degrees north latitude and 117.6793 degrees longitude, the closest major active faults to the site are the San Joaquin Hills Blind Thrust located 6.2 km southwest of the site, the Newport-Inglewood Fault (offshore) located approximately 21.3 km to the southwest of the site and the Whittier-Elsinore Fault located approximately 19 km north of the site.

Seismicity: Properties in southern California are subject to seismic hazards of varying degrees depending upon the proximity, degree of activity, and capability of nearby faults. These hazards can be primary (i.e., directly related to the energy release of an earthquake such as surface rupture and ground shaking) or secondary (i.e., related to the effect of earthquake energy on the physical world which can cause phenomena such as liquefaction and ground lurching). Since there are no known major or seismically active faults mapped at the site, the potential for primary ground rupture is considered slight to nil. The primary seismic hazard for this site is ground shaking due to a future earthquake on one of the major regional active faults, such as the San Joaquin Hills Blind Thrust, Newport-Inglewood, Whittier-Elsinore, San Andreas, and San Jacinto faults.

Site Class: Shear wave velocities (Vs) were obtained from two CPT probes at the site. CPT-1 was extended to a depth of 70 feet and CPT-4 was extended to a depth of 60 feet. The CPTs were probed in native alluvium. The CPT shear wave velocities were measured at approximate 10-foot-depth intervals from a depth of about 10 feet down to the total depth of the probe. This method automatically produces average velocity values for 10-foot-thick layers. The following table summarizes these layers with their corresponding shear wave velocities from these two CPTs.

Depth (ft.)	Shear Wave Velocity Vs (fps)	
	CPT-1	*CPT-4*
10 – 20	616	702
20 – 30	716	842
30 – 40	730	754
40 – 50	694	722
50 – 60	692	781
60 – 70	1075	--

These shear wave velocities represent the typical alluvium at the site in the upper 70 feet. These shear wave velocities indicate the onsite soils profile may be classified as Site Class D (stiff soil profile) per Table 1613.5.2 in the 2010 California Building Code.

Site Specific Seismic Hazard Analysis: A site-specific seismic evaluation has been performed in accordance with the methods described in the American Society of Civil Engineers (ASCE) Standard 7-05 and the 2010 CBC (Appendix D). This included determining the site class, selection of appropriate attenuation relationships, probabilistic analysis and deterministic analysis. The seismic (ground motion) hazard analyses were performed with the computer program EZFRISK, Version 7.52 (Risk Engineering, 2011) in conjunction with data from the US Geological Survey National Seismic Hazards Mapping Program (USGS, 2007).

Site Class and Attenuation Relationships: Based on the subsurface data, the subject site is classified as Site Class D (very stiff soil profile) from Table 1613.5.2 of the 2010 CBC. The regional attenuation relationships developed by Boore-Atkinson (2008), Campbell-Bozorgnia (2008) and Chiou-Youngs (2008) were used in our analyses. These Next Generation Attenuation (NGA) relationships were selected based upon their compatibility with the site-specific earth materials and seismic source conditions. The average spectral acceleration values from these attenuation relationships were used to generate the site-specific response spectra.

Probabilistic Analysis: Probabilistic seismic hazard analysis was performed to estimate the peak and spectral accelerations for the Maximum Considered Earthquake (MCE) ground motion values for active faults within a 60 mile radius. The above attenuation relationships were used for our probabilistic analysis. The probabilistic analysis was conducted for the MCE having a 2 percent chance of exceedance in 50 years, with a statistical return period of 2,475 years.

Deterministic Analysis: A deterministic seismic hazard analysis, assumed to attenuate to the site per the same attenuation relationships as the probabilistic method, was performed by evaluating the ground motions generated by maximum earthquakes on each of the active faults within the same search radius. Using this methodology, the maximum earthquake resulting in the highest peak horizontal accelerations at the site would be a magnitude 7.1 M_W event on the San Joaquin Hills Thrust Fault.

Site-Specific Design Response Spectra: For the site-specific analysis, the resultant accelerations were multiplied by 150 percent of the largest median 5 percent damped deterministic ground motions and compared to the results of the probabilistic analysis. The lesser of the probabilistic and larger of the 150 percent median deterministic and MCE deterministic lower limit spectrum is termed the Site-Specific MCE. The Site-Specific Design Response Spectrum is derived by taking 2/3 of the Site-Specific MCE spectral values (provided the results are not less than 80 percent of the Design Earthquake (DE) Response Spectrum).

Based on these USGS programs, the Controlling Fault for the subject site is the San Joaquin Hills Thrust Fault. The seismic hazard analysis and graphs, including probabilistic and deterministic spectra and the final site-specific design response spectrum, are presented in Appendix D. The recommended site-specific seismic design parameters are tabulated in Section 3.12 of this report.

Secondary Seismic Hazards: The majority of the site is mapped by the State of California in seismic hazard zones for potential liquefaction (CDMG, 2001). The subject site location in relation to the potentially liquefiable zones is shown on Figure 1. The potential liquefaction hazard is discussed in Section 2.7 Secondary seismic hazards such as tsunami and seiche need not be considered, as the site is located away from the ocean or confined bodies of water.

We understand that the County of Orange Public Works policy is not to design at grade roadways and other improvements for secondary seismic hazards (Kleinfelder, 2009). In keeping with that policy, NMG has not included detailed analyses or discussion related to the potential impacts of secondary seismic hazards (particularly liquefaction effects on the box culvert. Some discussion is provided for informational purposes only. Our focus in this report with respect to evaluating and mitigating secondary seismic hazards is on the potential impacts to the adjacent residential developments and slopes that support it.

2.5 Surface Water and Groundwater

Surface water flows year round within the northernmost portion of the Borrego Wash. The annual flow is from an offsite storm drain that connects to the residential and commercial developments to the north of the site. The northern most portion of the active wash consists of exposed sandstone bedrock. Periodic heavy storm runoff has cut an incised channel into the sandstone creating a minor canyon with some topographic steps that result in minor waterfall conditions. Further downstream the wash is underlain by relatively permeable alluvium and the water infiltrates underground except during the winter rainy season.

Groundwater and/or seepage was encountered in the majority of the geotechnical borings excavated for this study. Groundwater was also observed in many of the prior borings and other excavations. Groundwater was encountered within the alluvium and, where the alluvium and bedrock contact is shallow, was perched at the lithologic contact between the two earth units. Because of the different years and different times of year that exploratory excavations were made, the groundwater level below the Borrego flood plain and wash appears to fluctuate substantially. However, analysis of the more recent excavations compared to our recent subsurface exploration indicates that the groundwater has most likely stabilized and only fluctuates a few feet between the summer and winter months. It is NMG's opinion that this relative consistency is because there is a constant source of water, via the storm drains, from developments to the north of the site. Due to the constant source of surface runoff from the Foothill Ranch area, the present groundwater level is considered the historic high for this area.

In general, in the area of the flood plain and wash, the groundwater is anticipated to be within 5 to 10 feet below the level of the active Borrego Wash, with the depth to groundwater from ground surface increasing to the southwest. It is anticipated that this groundwater level also affects the level of groundwater in the larger side canyons that connect into the Borrego Wash and Plain.

2.6 Mass Movements

There are local small areas of bluff failure and surficial erosion along the lateral margins of Borrego Wash. There are no landslides mapped at the site, and landslides were not encountered during previous investigations or grading at the site (PSE, 2002). Also, based on the seismic hazard mapping by the State (CDMG, 2001), areas of potential seismically induced landslides are not mapped within the subject site (Figure 1).

2.7 Liquefaction Potential

General: Liquefaction is a phenomenon in which earthquake-induced cyclic stresses generate excess pore-water pressure in low density (loose), saturated, sandy soils and soft silts below the water table. This causes a loss of shear strength and, in many cases, ground settlement. For liquefaction to occur, all of the following four conditions must be present:

- There must be severe ground shaking, such as occurs during a strong earthquake.
- The soil material must be saturated or nearly saturated, generally below the water table.
- The corrected normalized standard penetration test (SPT) blow counts (N_1) or the CPT tip resistance (Q) must be relatively low.
- The soil material must be granular (usually sands or silts) with, at most, only low plasticity. Clayey soils and silts of relatively high plasticity are generally not subject to liquefaction.

There are four possible adverse consequences of liquefaction of sandy soil layers that are addressed below:

- Liquefaction-induced settlements;
- Loss of bearing and other possible local disruptions at the ground surface (sand boils);
- Lateral spreading; and
- Global slope instability due to flow liquefaction.

Exploration Analysis: The liquefaction potential at the site was assessed based on 7 CPTs (CPT-1 through CPT -7). The nearby hollow-stem-auger borings and sampling by a CPT support rig were utilized to verify the empirical soil material descriptions presented in the CPT logs.

Our liquefaction potential assessment was performed using the computer program CLiq version 1.5 developed by Geologismiki which provides results and plots of the calculations. The liquefaction potential analysis is performed using the Robertson 2009 method. The soil type behavior estimations are based on Robertson, P.K. 1990. The program provides the basic CPT data interpretation through to final plots of factor of safety, liquefaction potential index and post-earthquake displacements, and settlement.

The liquefaction potential of the onsite soils were estimated based on a deterministic site acceleration of 0.35 g and a maximum earthquake magnitude of 7.1 as determined in our site seismicity analysis discussed in Section 2.4.

Based on the results of the calculated data, the liquefaction potential at the site is considered low to moderate. In general, the potentially liquefiable layers consist of alluvial/colluvium that generally range from 0.5 to 2.5 feet thick and locally up to 6 feet.

Seismic Settlement: The results of our analysis indicate that the liquefiable layers in the alluvium, when subjected to the high ground accelerations of a large earthquake event near the site, will be subject to settlement. Based on our calculations, the settlement due to liquefaction is anticipated to range from less than one inch to greater than 3.2 inches for the residential development areas (above the fill slope) and to up to 6.4 inches for the culvert box.

Loss of Bearing: The potential for loss of bearing was reviewed based on the thickness of the liquefiable layers that will be left in place, versus the amount of fill and non-liquefiable alluvium that will overlie the liquefiable soils. Local surface disruptions and loss of bearing strength at the surface are unlikely because the potentially liquefiable lenses are relatively thin and will be overlain by thicker, non-liquefiable material within the building sites after the grading is completed.

Lateral Spread/ Liquefaction Potential Index: In evaluating the potential for lateral spreading at the subject site, we have reviewed the readily available research based on empirical data regarding lateral spreading potential due to liquefaction of soils, and evaluated the general soil stratigraphy at the site.

Studies have been performed at sites that experienced lateral displacement/spreading along with adjacent sites that did not report any damages during recent earthquakes (Toprak and Holzer, 2003, Holzer, et al., 2003 and 2011 and Papathanassiou, 2008). These studies applied the Liquefaction Potential Index (LPI) originally proposed by Iwasaki (1978), at the different sites. LPI for the entire soil column at those locations were calculated. Based on findings presented in these referenced studies, the median and lower quartile values of LPI for occurrence of lateral spreading are 12 and 5, respectively. The LPIs calculated by the computer program used in our analysis for the subject (Appendix E) are lower than 5. This indicates that the potential for lateral spread is low.

As previously discussed, the majority of the liquefiable layers are thin and on the order of less than 2 feet. Also, layers are typically not continuous across the site. Based on our review of the collected data discussed above, we conclude that the potential for lateral spreading at the site is considered low.

Flow Liquefaction: Although the potential for large scale lateral spreading at the site is low, the potential for local flow-type failures adjacent to the Borrego Wash, due to loss of liquefied soil strengths following a large seismic event near the site, cannot be ruled out. This potential for flow-type failures is discussed in the following section.

2.8 Slope Stability

Design Fill Slopes: There are planned fill slopes of up to approximately 55 feet in total height within the site, with a mid-slope bench which is generally over 40 feet in width. Design fill slopes descending from the mid-slope bench to Borrego Wash are up to 30 feet high and range from 0.3H:1V to 2H:1V, with the portions steeper than 2H:1V (approximate RCB Stations 41+50 to 52+00) shown on the improvement plans to be treated with soil cement to potentially increase strength.

For this report, we analyzed three cross-sections (A-A', C-C'; and D-D') representative of the proposed design slope conditions along the Borrego Wash. Soil strength parameters for use in the slope stability analyses were derived from data and reports by Hushmand (2010), PSE (2002) and our recent laboratory testing. The parameters along with a description of the software used, methodology, and results of our analyses are included in Appendix F.

Our analyses show that the planned fill slopes should be grossly stable with factors of safety of 1.5 and 1.1 or greater for static and seismic cases, respectively.

Natural Slopes: The natural slopes on the northwest side of the active Borrego Wash have been impacted by erosion and have become undercut since the early 1990s. At that time, runoff from Foothill Ranch became channelized and surface flows increased. The erosional impacts include incised cuts into sandstone bedrock (Stations 45 through 50), and substantial down-cutting and widening of the wash to the south end of the property at the outlet structure. The bypass channel is designed to collect the majority of the surface runoff from the existing channel and only allow low flow surface runoff to continue down Borrego Wash (Hunsaker, 2011).

Flow Liquefaction: The total design fill slope along the Borrego Wash is up to 55 feet high. The fill slope along the wash may be subject to deformation or failures following a large seismic event near the site. The high ground accelerations may liquefy some layers and cause a loss of shear strength in the subsurface soils. In order to help mitigate the effects of flow liquefaction, a stabilization fill key is recommended at the toe of slope (See Sections 3.4 and 3.5).

Surficial Slope Stability: Surficial slope stability was also evaluated based on infinite slope stability analysis. The surficial stability depends upon the steepness of the slopes and the compaction and strength of near-surface soils (upper 4± feet). The onsite soils are anticipated to consist of generally silty sandy materials. The design cohesive strength of 100 psf used for surficial slope stability of 2H:1V slopes results in a factor of safety of 1.12. The sandy composition of the embankment indicates that the slopes may not be surficially stable if sandy materials are used within the upper 4 feet. At the north end of the project, there are slopes designed steeper than 2H:1V and are shown on the plans as being treated with soil cement. NMG is providing alternate recommendations for slope design and did not analyze the soil cement design. Recommendations related to surficial slope stability of the proposed slopes are provided in Section 3.5.

2.9 Static Settlement

General Fill Areas: Based upon previous subsurface exploration, laboratory testing and analysis, significant amounts of alluvium, as well as the minor amounts of colluvium and undocumented fill may be prone to significant collapse and/or consolidation and have poor bearing properties. The thickness of this unsuitable soil zone varies from approximately 2 to 40 feet across the site. Below these materials the un-weathered terrace deposits and bedrock materials have favorable properties with respect to bearing capacity and settlement potential.

Throughout the majority of the site, new fills up to 40 feet in depth are being proposed. The amount of potential settlement can vary significantly over the site due to variations in subsurface conditions and depths of planned cuts and fills. In conducting settlement analyses, we have assumed that remedial removals will remove the unsuitable soils to saturated alluvium which is anticipated to be left in place.

The total thickness of designed fills and fills as a result of remedial removals may locally exceed 60 feet in depth. We anticipate that maximum total settlements at the site will be on the order of multiple inches over a period of 50 years, but within typically accepted tolerances for the proposed development, provided some time elapses following the completion of grading. Maximum settlement waiting periods where some amount of the primary settlement is allowed to take place are generally expected to be on the order of 1 to 6 months or more, depending on the amount of new fill and the earth materials beneath.

Box Culvert: Replacement of unsuitable soil with denser compacted fill together with raising grades above existing ground along the box culvert alignment will induce varying amounts of settlement below the culvert. More detailed settlement analyses were conducted along the box culvert alignment to evaluate their magnitude and potential impact on the culvert. Consolidation parameters for these analyses were derived from laboratory testing by NMG, review of subject site data by others (Kleinfelder, 2009; Hushmand, 2010; PSE, 2002), and our experience with similar soils in the area.

Cross-sections A-A' through F-F' were analyzed for static settlement, taking into account the dimensions and weight of the box culvert, existing ground elevations, and the proposed design grades above the box and adjacent areas including the larger fill slopes for the adjacent residential areas. Existing and planned grades and the box alignment were determined from the grading plan by Hunsaker and Associates.

Calculated total static settlements along the majority of the alignment were on the order of 1 inch or less. However, one segment, from Station 19+00 to 24+00 (500 feet) had calculated settlements of approximately 2¼ inches (represented by Cross-section E-E'). The larger settlements are due to a combination of greater fill loads above the box (on the order of 9 feet) and deeper alluvium below the box that will be left in place. Mitigation of this condition is discussed in Section 3.2.2.

2.10 Earthwork Shrinkage/Bulking and Subsidence

The loss or gain of volume (shrinkage or bulking, respectively) of excavated natural materials and re-compaction as fill, varies according to earth material type and location. This volume change is represented as a percentage shrinkage (volume loss) and as a percentage bulking (volume gain) after re-compaction of a unit volume of cut in this same material in its natural state. The onsite materials will have varying shrinkage or bulking characteristics. The following table presents the projected range of values for each type of material:

Earth Unit	*Approximate Percent Shrinkage/Bulking*
Existing artificial fill and alluvium/colluvium	5 to 15 percent shrinkage
Terrace Deposits	0 to 5 percent shrinkage
Capistrano Formation	0 to 4 percent bulking

Ground subsidence at the site is estimated to be on the order of 0.1 foot across the site.

2.11 Existing Utilities

There are many agricultural irrigation pipelines that cross the property. There are also several existing buried and above ground utilities that service the existing nursery facilities at the site. We assume that existing septic systems and cesspools may exist near the buildings/sheds onsite.

2.12 Rippability and Generation of Oversize Material

The rippability characteristics of bedrock depend upon the rock type, hardness, the depth of weathering, degree of fracturing, and the structure of the rock. Based on the reviewed improvement plans, only minor amounts of bedrock are anticipated to be excavated during construction of the box culvert, drainage improvements, and adjacent slope construction.

Borings excavated throughout the site using bucket augers and other forms of drilling were excavated to a maximum depth of 80 feet into the bedrock and earth material without refusal. The equipment used to excavate these borings typically cannot excavate, without coring, earth materials that are not rippable.

Based on prior explorations and grading within the site, the bedrock should be rippable with D-9 and D-10 bulldozers. Proper equipment selection and sound ripping techniques are important for effective earthwork operations. NMG anticipates that only a minor amount of oversize rock (greater than 12 inches in the maximum dimension) will be generated from localized cemented zones within the bedrock.

3.0 CONCLUSION AND PRELIMINARY RECOMMENDATIONS

3.1 General Conclusion and Recommendation

Based on our findings, the site is considered geotechnically feasible for the proposed Borrego Wash Improvements and Bypass Channel, and adjacent residential development provided the recommendations of this report are implemented during grading and future design and construction. It is our opinion that the bypass channel and planned grading provide substantial improvement to the active Borrego Wash, including the adjoining natural slopes. Our recommendations may be superseded by more stringent requirements of the governing agency or other members of the design team. The grading and construction should be performed in accordance with the City of Lake Forest Grading Code and the grading specifications provided in Appendix G, except as superseded below.

3.2 Remedial Grading

Substantial remedial removals are anticipated to bring the site to structural conditions as shown on the tentative tract map. Demolition of existing site improvements associated with prior land use will be required during remedial grading at the site. These improvements include existing utilities, nursery structures, onsite sewage disposal structures (if any), abandoned storm drain segments, drainage basins, etc. Depths of the demolition and remedial removals are provided below.

3.2.1 Demolition

Foundations associated with the existing nursery structures, drainage devices, windmill, temporary erosion-control devices, etc., shall be demolished and removed from the site during remedial grading. Demolition will include removal of existing nursery water pipelines, overhead electrical poles/lines and temporary drainage devices.

Based on our understanding, there may be old septic systems for the nursery at the site. If encountered, the septic systems should be removed during grading.

3.2.2 Remedial Removals

Unsuitable earth materials should be removed prior to placement of proposed fill. Unsuitable materials at the site include topsoil, alluvium, colluvium, undocumented fills, the weathered terrace deposits, and weathered bedrock. Estimated removal depths are fairly consistent across the site.

Undocumented fills associated with farming operations, detention/desilting basins, the network of unpaved access roads, old drainage channel infill, and existing trench backfills should be removed prior to fill placement. Generally, these artificial fills range in depth from 3 to 10 feet in thickness.

Unsaturated alluvial/colluvial material should be removed prior to fill placement. The unsaturated portions of these deposits are anticipated to range from 10 to 40 feet in depth across the site. Saturated alluvium/colluvium (having a minimum 85 percent degree of saturation) deposits are anticipated at the removal bottom. These saturated deposits may be left in place, provided the settlement and time delay consequences are acceptable by the project owner.

Removal bottoms exposing competent material should be evaluated and accepted by the geotechnical consultant. The removal bottoms should be scarified, moisture-conditioned and recompacted prior to placement of compacted fill unless the removal bottom consists of saturated material. Where removal bottoms expose saturated material, bridging with gravels, sands, and/or geofabric may be necessary locally for workability. These areas will need specific evaluation based on the actual conditions at the time of grading and the planned thickness of overlying fill.

3.2.3 Box Culvert Overexcavation and Fill Blanket

To help mitigate settlement and provide uniformity beneath the box culvert, we recommend a minimum of 8 feet of compacted fill be placed below the box. In some places, the design fill achieves this condition. Were it does not achieve this condition, we recommend overexcavation and replacement with compacted fill. The overexcavation should be extended at a 1H:1V projection from the outside corner of the box as shown on the cross-sections (Plates 5 and 6). Locally, this overexcavation encompassed by the seismic shear key. For some areas, this will require dewatering and removals below the water table. Granular fill should be placed in the special removal area and should be placed at a minimum of 93 percent relative compaction. Fill above the level of the culvert bottom may be placed at 90 percent relative compaction. At the north end of the project (Station 48±), the excavation for the box should expose sandstone bedrock. In this area, overexcavation is not necessary.

3.2.4 Inlet and Outlet Structures

The inlet and outlet structures are anticipated to have alluvium exposed at subgrade. We recommend an 8 foot over-excavation of the relatively loose earth materials and replacement with compacted fill to be performed to provide a competent subgrade to construct the improvements. The compaction should be performed in accordance with the project specifications.

3.3 General Earthwork and Grading

Prior to commencement of grading operations, deleterious material (including highly organic topsoil, vegetation, trash, unsuitable debris) should be cleared from the site and disposed of offsite. Numerous irrigation lines are anticipated that cross the site. These lines should be removed and the areas should be properly backfilled if determined to be below the removal bottom.

Grading and excavations should be performed in accordance with the City of Lake Forest Grading Code and the General Earthwork and Grading Specifications in Appendix G. Prior to placement of fill, removal bottoms should be scarified a minimum of 6 inches, moisture-conditioned as needed, and compacted to a minimum 90 percent relative compaction. Fill material should be placed in loose lifts no greater than 8 inches in thickness and compacted prior to placement of the next lift. Ground sloping greater than 5H:1V should be prepared by benching into firm, competent material as fill is placed. Relative compaction should be based upon ASTM Test Method D1557. Moisture content of fill soil should be over optimum moisture content.

Native materials that are relatively free of deleterious material should be suitable for use as compacted fill. If import soils are required in order to achieve design grades, they should be evaluated by the geotechnical consultant prior to and during transport to the site to verify their suitability. Wet soils may require drying back prior to placement as fill. Saturated remedial removals below the groundwater table are expected during construction of the seismic shear key discussed in Section 3.4.1.

Removal bottoms, seismic shear key location, backcuts, and canyon subdrains should be surveyed prior to observation, mapping and acceptance by the geotechnical consultant.

3.4 Slope Stabilization

Fill slopes up to 55 feet in height with a mid-slope bench which is generally 40 feet in width are planned along the Borrego Wash. In general, the keys and excavations should be evaluated and accepted by the geotechnical consultant prior to placement of a canyon-type subdrain and/or backfill.

3.4.1 Proposed Fill Slopes

There is a fill slope up to 55 feet in total height planned along the entire length of the development adjacent to Borrego Wash. A seismic shear key is recommended generally along the RCB alignment to mitigate potential flow failure due to liquefaction during the design earthquake event (see Appendix F for further discussions). The recommended shear key extends from approximately RCB Station 21+00 to 45+00. The seismic shear key is designed at 40 feet wide and 12 to 15 feet deep below the recommended remedial removal bottoms. The location and dimensions of this shear key are shown on Plates 1 through 4. Cross-Sections A-A, C-C' and D-D' (Plates 5 and 6) show the approximate configuration of the recommended seismic shear key and the remedial removals adjacent to the key. The shear key should be constructed in accordance with our Grading and Earthwork Specifications (Appendix G).

Locally, the proposed fill slope along the wash is designed steeper than 2H:1V (approximate RCB Stations 41+50 to 52+00) with soil cement enhancement. NMG recommends these slopes to be redesigned as 1½:1 geogrid reinforced slopes or a plantable mechanically stabilized earth retaining wall (e.g., Verdura wall).

Some of the onsite materials that will be used for fill are clean sands with very little cohesion. We recommended that the fill materials used for the outer 15 feet of any fill slope be constructed with earth materials that are more cohesive to help reduce the potential for erosion from rain (on the order of 10 percent passing the No. 200 sieve). If more cohesive earth materials are not available, then finished slopes should be protected with spray-on protective coverings, jute matting, and/or other special erosion control measures until slope ground cover vegetation can be sufficiently established (with deeper rooted plants).

These fill slopes are anticipated to be stable as designed provided they are constructed in accordance with the details in our General Earthwork and Grading Specifications (Appendix G).

3.4.2 Temporary Stability

Temporary slopes will be created by the backcuts for the recommended remedial removals and seismic shear key. Backcuts are designed with a slope ratio of 1H:1V and are up to 15 feet in height. Temporary slopes for the shear key may be below the anticipated groundwater table. The actual stability of the backcuts will depend on many factors, including exposed earth materials, amount of unloading performed prior to backcut excavation, and amount of time the excavation remains exposed. Proper remedial measures should be provided to protect the adjacent properties in-place. Measures to mitigate potential backcut failure may include the following:

- The excavation bottoms should not be left open for long periods of time; the lower portions of the key should be backfilled as soon as practical (i.e., backfilled prior to the weekend if possible).
- The backcut and front cut should be carefully excavated at the recommended slope angles and "on grade" to reduce oversteepened areas. Cutting areas at steeper angles may result in slope failure.
- If necessary, the keyway and remedial grading operations may need to be constructed in sections (on the order of 100 feet long); shorter sections may be necessary if backcut failures occur.

3.4.3 Natural Slopes

There are natural slopes located along the western edge of the project immediately adjacent to the west side of Borrego Wash. The slopes along the majority of Borrego Wash range from 2H:1V to near vertical. This condition is the result of active creek erosion. Although these slopes are outside the grading limits, they are marginally stable and may contribute sediment to Borrego Wash due to future erosion and shallow slumping. Following the proposed improvements, Borrego Wash will transmit low flow runoff only which will reduce the erosive forces on the offsite natural channel. Even with this favorable change, periodic maintenance may be required to clean out the natural creek area to maintain the channel shape.

3.5 Erosion Mitigation

The majority of the future surface runoff from the northerly (offsite) section of Borrego Wash will be channeled into the box culvert planned along the western edge of the Baker Ranch development. The low flows will be collected in a 60-inch-diameter RCP and directed into the present channel for Borrego Wash. The low flow anticipated is 50 to 200 cfs (Hunsaker, 2011). Although this amount/velocity of runoff is not considered erosive, the civil engineer has designed rip rap slope protection at the toe of slope which daylights in the bottom of Borrego Wash. The rip rap covers the lower 4 feet of the 3H:1V slope and extends at the same angle such that the end the rip rap is 5 feet below the channel bottom. The subgrade for this protection will be engineered fill and should be compacted to 90 percent relative compaction in accordance with project specifications.

3.6 Groundwater

The groundwater surface at the site has been encountered at depths within 5 to 10 feet below the level of the active Borrego Wash, with the depth to groundwater from ground surface increasing to the southwest. The recommended seismic shear key and locally, the recommended remedial removals, will encounter groundwater.

3.7 Dewatering

Dewatering should be anticipated to facilitate grading and construction. We anticipate that the excavations for the seismic shear key and locally, remedial removal excavations will expose loose saturated poorly graded sand and static groundwater conditions. The recommended seismic shear key extends approximately 12 to 15 feet below the groundwater table.

Based on prior experience, personal communication and observation at the adjacent rough grading operation for Alton Parkway, this condition has been successfully mitigated by dewatering directly from the excavation with a well-point system and/or overexcavated areas plated with gravel and using portable pumps. Local areas may require temporary shoring to mitigate caving. We believe similar measures can be utilized for the recommended rough grading. However, the contractor should evaluate the anticipated conditions and provide groundwater mitigation measures based on their experience and expertise.

A qualified dewatering specialist should be retained to design the appropriate dewatering system.

3.8 Stabilization of Wet Removal/Shear Key Bottoms

If the removal and seismic shear key bottoms become too wet due to shallow groundwater conditions, it may be necessary to place select gravel material to stabilize the removal bottoms. Wet removal excavations will likely need to be performed with an excavator or special earthwork equipment. Where wet material is exposed that is considered too saturated to support compaction equipment required to produce a certified fill, stabilization of the removal bottom will be required. Stabilization options may include placement of a geotextile product (Mirafi 160 N or approved equivalent) across the entire wet area with a minimum overlap of 18 inches (at product edges) and/or placing a minimum 12 to 24 inches of crushed aggregate across the shear key/removal bottom.

3.9 Subdrainage

Canyon-type subdrains (9-cubic-feet-per-foot of gravel with a 6-inch, Schedule 40, perforated pipe wrapped in filter fabric) are recommended at the alluvium-bedrock interface on the removal bottom prior to placement of fill. These subdrains should be provided with a suitable outlet. Where the canyons are wider, the need for additional subdrains should be evaluated by the geotechnical consultant during grading. The details for subdrains are included in our Earthwork and Grading Specifications (Appendix G).

3.10 Settlement

Settlement: Recommended remedial removals (Section 3.2) are intended to remove the potentially collapsible and/or very compressible near-surface unsaturated alluvial material. The amount of settlement where saturated alluvium will be left in place will depend on the thickness of the alluvium and design fills and loading conditions.

We recommend that the deeper fill areas (greater than 40 feet) and areas where more than 10 feet of fill will be placed over relatively thick older fill and/or left-in-place alluvium be monitored for settlement with a combination of buried settlement plates and surface monuments. The location of these devices should be determined at the 40-scale grading plan review stage. Installation of the devices is typically the task of the geotechnical consultant during and at the completion of grading. Surveying of the devices at the time of installation and subsequent monitoring should be performed by a licensed surveyor.

Fills deeper than approximately 50 feet below finish grade should be compacted to a minimum of 93 percent relative compaction to reduce the amount and time related to long-term settlement.

The frequency of settlement monitoring (survey readings) will depend upon the grading and construction schedule and other factors, such as the timing of residential building and occupancy. Construction of structures should not commence until the geotechnical consultant has determined, from settlement monitoring, that remaining settlements are within acceptable limits for the intended improvements.

The settlement associated with potential liquefaction of the granular soils at the site is estimated to range from less than 1 inch up to approximately 3 inches for the residential development areas (above the fill slope). The potential settlements are within the typical tolerance of residential improvements. Details of our analysis are presented in the Appendix E.

3.11 Box Culvert Settlement/Surcharge

It is our understanding that the differential settlement tolerance for the box culvert may be as small as ½ inch over 24 feet (Kleinfelder, 2009). Special recommendations for grading are needed to mitigate this settlement tolerance. Between approximate Stations 19+76 to 24+00, the fill planned adjacent to and above the culvert should be placed to final design grades prior to construction of the box culvert. This fill should remain entirely in place for at least four weeks before the excavation is performed to facilitate construction of the box culvert.

3.12 Liquefaction

Slope failures following large seismic events to due liquefaction will be mitigated by the recommended shear key (Section 3.4.1). The potential for surface manifestation (i.e. sand boils) caused by liquefaction is considered slight since there will be sufficient amount of non-liquefiable materials (fill and alluvium) overlying the buried liquefiable layers. The potential for large lateral spreads is considered low due to the discontinuous nature of the alluvium underlying the site and the LPI calculated for the site soils (see Section 2.7).

3.13 Seismic Design Parameters

The following table summarizes the seismic design criteria for the subject site. The site-specific design response spectrum was developed in accordance with 2010 CBC and Chapters 11 and 28 of ASCE Standard 7-05.

Selected Seismic Design Parameters	*Seismic Design Values*
Latitude	33.6743 North
Longitude	117.6793 West
Controlling seismic source	San Joaquin Hills Thrust Fault
Distance to controlling seismic source	6.2 km (3.9 miles)
Site Class per Table 1613A.5.2	D
Maximum Considered Earthquake spectral response acceleration for short periods (S_{MS}) from site-specific analysis (Site Class D)	1.12g
Maximum Considered Earthquake spectral response acceleration for 1-second periods (S_{M1}) from site-specific analysis (Site Class D)	0.87g
Five-percent damped design spectral response acceleration at short periods (S_{DS}) from site-specific analysis (Site Class D)	0.75g
Five-percent damped design spectral response acceleration at 1-second period (S_{D1}) from site-specific analysis (Site Class D)	0.58g

The peak ground acceleration based on site-specific analysis is 0.35g (Appendix D). This is the acceleration utilized in the liquefaction analysis.

3.14 Lateral Earth Pressures

Based on laboratory test results and our previous experience on similar projects, we recommend the following lateral earth pressures for native soils in drained conditions:

Conditions	Equivalent Fluid Pressure (psf/ft.)	
	Level	*2:1 Slope*
Active	40	65
At-Rest	60	85
Passive	350	130 (sloping down)

In addition to the above lateral forces due to retained earth, the influence of surcharge due to other loads such as adjacent footings, or lateral load acting on screen walls above the retaining wall, if any, should be considered during design of retaining walls.

To design an unrestrained retaining wall, such as a cantilever wall, the active earth pressure may be used. For a restrained retaining wall, such as a basement wall, or at restrained wall corners, the at-rest pressure should be used. Passive pressure is used to compute lateral soil resistance developed against lateral structural movement. Further, for sliding resistance, the friction coefficient of 0.35 may be used at the concrete and soil interface. In combining the total lateral resistance, either the passive pressure or the frictional resistance should be reduced by 50 percent. In addition, the passive resistance is taken into account only if it is ensured that the soil against embedded structures will remain intact with time.

In areas where remedial removals may not be possible to provide competent conditions under the footings, the retaining walls should be placed on a deepened footing that extends down into competent native soils. This may be accomplished by either deepening the conventional cantilever footing or providing the wall with caisson and grade beam foundation.

The seismic lateral earth pressure for the level backfill and using a seismic coefficient of 0.15 may be estimated to be an additional 14 pcf for active and at-rest conditions. The earthquake soil pressure has an inverted triangular distribution and is added to the static pressures. For the active and at-rest conditions, the additional earthquake loading is zero at the base and maximum at the top.

3.15 Bearing Capacity and Modulus of Subgrade Reaction

For the anticipated subgrade soils, allowable bearing capacity at the site may be calculated with the following formula:

$$Q_a = 2{,}000 + 400D + 200B,$$

to a maximum of 4,000 psf for the box culvert; (where D is the foundation embedment below competent grade and B is the foundation width). The maximum allowable bearing capacity for other structures is typically 3,000 psf but may depend on their location and sensitivity to settlement. The bearing capacity may be increased by on third for seismic or wind loads. The formula is based on a design groundwater table 5 feet below bottom of foundation/culvert.

A modulus of subgrade reaction of 20 pci may be assumed for the box culvert founded on soil graded in accordance with the recommendations in this report.

3.16 Structural Setbacks

The footings of structures located above descending slopes should be set back from the slope face in accordance with the minimum requirements of the City of Lake Forest and CBC criteria, whichever is greater. The setback distance is measured from the outside edge of the footing bottom along a horizontal line to the face of the slope. For the subject site, the maximum descending slope height is approximately 55 feet.

The table below summarizes the minimum setback criteria for structures above descending slopes:

Structural Setback Requirements for Footings Above Descending Slopes

Slope Height [H] (feet)	*Minimum Setback from Slope face (feet)*
Less than 10	5
10 to 20	½ * H
20 to 30	10
More than 30	⅓ * H (maximum of 40')

Additional consideration and recommendations for top-of-slope walls (freestanding) or other improvements that are sensitive to lateral movement will be provided in our report for the residential development.

3.17 Rippability and Placement of Oversize Material

The bedrock at the site includes portions that are dense cemented sandstone that may be difficult to rip. We anticipate that the bedrock will be rippable in the planned excavations (due to the shallow design cuts and minimal remedial excavations into the bedrock material).

Local excavations within the bedrock cuts may produce oversize rock (greater than 12 inches in size) that will require special placement in the fill. Oversize rock may be placed in fills deeper than 10 feet, and a minimum of 2 feet below the deepest utilities within the streets. Placement of oversize material should be performed in accordance with our General Earthwork and Grading Specifications in Appendix G. Grading operations should be carefully planned so that the fills deeper than 10 feet can accept oversize rock from the cuts.

3.18 Expansion Potential

The expansion potential of site soils is generally anticipated to range from "very low" to "low" per ASTM D4829 classification. Although some relatively thin clayey siltstone and claystone beds could be of very high expansion potential. At the completion of grading operations, soil samples should be collected at finish grade and tested for expansion potentials to confirm anticipated conditions.

3.19 Concrete in Contact with Soil

The soluble sulfate content for the onsite alluvial soils is within the range of "negligible sulfate exposure" for concrete as classified in Table 4.3.1 of ACI-318. Although the ACI does not require any special concrete design for "negligible sulfate exposure," we recommend that, as a minimum, Type II cement be used even with negligible sulfate exposure. Moreover, we recommend that additional sulfate testing be performed at the site on soils exposed at the surface after grading is complete.

3.20 Surface Drainage

Surface drainage should be carefully taken into consideration during all grading, landscaping, and building construction. Positive surface drainage should be provided to direct surface water away from structures and slopes and toward the street or suitable drainage devices. Ponding of water adjacent to the structures should not be allowed. Paved areas should be provided with adequate drainage devices, gradients, and curbing to reduce run-off flowing from paved areas onto adjacent unpaved areas.

3.21 Maintenance of Graded Slopes

To reduce the erosion and slumping potential of the graded slopes, all permanent manufactured slopes should be protected from erosion by planting with appropriate vegetation or suitable erosion protection should be applied as soon as is practical. Proper drainage should be designed and maintained to collect surface waters and direct them away from slopes. The maintenance program should take into account the granular, more erodible nature of the soils that are likely to be present at the slope face at the completion of grading. Consideration should be given to the use of spray-on protective products and frequent use of straw waddles or other temporary runoff control devices immediately after slopes are constructed. In addition, the design and construction of permanent improvements and landscaping should also provide appropriate mitigation measures for sandy soils. A rodent-control program should be established and maintained as well, to reduce the potential for damage related to burrowing.

3.22 Utility Construction

Shoring: RCB excavations should be stabilized per OSHA requirements (shoring or laying back of trench walls) for Type B soils within certified engineered fill and locally for Type C soils due to possible adverse bedding conditions or loose, running sands.

Pipe Bedding and Sand Backfill: Pipe should be placed on at least 6 inches of clean sand or gravel. The area around the pipe (at least one foot over top of pipe) should be backfilled with clean sand, having a minimum sand equivalent (SE) of 30 or better. The sand could be jetted with water below the springline to ensure filling of voids beneath the pipe (if allowed by local agency). Otherwise, sand along the side of the pipe should be placed in small lifts and compacted with small hand-held compactors (e.g., powder-puffs). Depending on the size of the pipe, higher sand equivalents may be required if jetting is not permitted. Jetting should be performed in moderation to minimize the amount of water introduced into the surrounding native soils.

Trench Backfill: Backfill materials should be moisture-conditioned as needed to within the compactable range and compacted to a minimum relative compaction of 90 percent. Some oversize rocks may be generated from the cuttings from the trenches, if the streets and lots are not overexcavated. These oversize rocks will need to broken down in size or exported from the site. It is anticipated that rocks less than 6 inches in the maximum diameter can be placed in the backfill 2 feet above the pipe zone and 2 feet below subgrade.

Remedial earthwork within the subject site will encounter significant quantities of sand that has a sand equivalent (SE) of 30 or greater, and therefore, may be suitable for structural backfill. Because there are occasional silty layers mixed in with the sand, potential source areas should be evaluated by the geologist/engineer prior to use.

3.23 Geotechnical Review of Future Plans

Future revisions/changes to the current grading plan and storm drain improvements for the proposed site development should be reviewed and accepted by the geotechnical consultant prior to grading. The additional improvement and future 40-scale rough grading plans for the site and adjacent areas should also be reviewed to evaluate the potential impacts to the site and provide specific details for grading and construction. Additional geotechnical reports with recommendations specific to the construction improvements should be provided once plans are available.

3.24 Geotechnical Observation and Testing During Grading

The findings, conclusions and recommendations in this report are based upon interpretation of data and data points having limited spatial extent. Verification and refinement of actual geotechnical conditions during grading is also essential, especially where slope stabilization is involved. At minimum, geotechnical observation and testing should be conducted during grading operations at the following stages:

- During and following clearing and grubbing, prior to site processing;
- During demolition of existing structures, foundations or other existing site improvements;
- During and following remedial removals to evaluate the removal bottom;
- During and following cutting of slopes and excavation of slope stabilization measures;
- During installation of subdrains;
- During placement of compacted fill;
- During construction of utility lines (if applicable);

- During and upon completion of excavations for storm drain structures and during trench backfill; and
- When any unusual or unexpected geotechnical conditions are encountered during grading and construction.

4.0 LIMITATIONS

This report has been prepared for the exclusive use of our client, Shea Homes, based on the specific scope of services requested by Shea Homes for the Baker Ranch project described herein. This report or its contents should not be used or relied upon for other projects or by other parties without the consent of NMG and the involvement of a geotechnical professional. The means and methods used by NMG for this study are based in part on local geotechnical standards of practice, care, and requirements of governing agencies. No warranty or guarantee, express or implied is given.

The findings, conclusions, and recommendations are professional opinions based on interpretations and inferences made from geologic and engineering data from specific locations and depths, observed or collected at a given time. By nature, geologic conditions can vary from point to point, can be very different in between points, and can also change over time. Grading and other project plans also are still being developed. Therefore, our conclusions and recommendations are by nature preliminary and are subject to verification and possible modification as plans develop.

Inherently, geotechnical recommendations are also preliminary until the geotechnical consultant observes and tests exposed subsurface conditions during grading and construction. The recommendations in place at that time are subject to modification at the discretion of the geotechnical consultant depending upon exposed geotechnical conditions.